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IS : 11682 - 1985
(Reaffirmed 1991)

Indian Standard

**CRITERIA FOR
DESIGN OF RCC STAGING FOR
OVERHEAD WATER TANKS**

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BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
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Indian Standard

CRITERIA FOR DESIGN OF RCC STAGING FOR OVERHEAD WATER TANKS

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Indian Standard

CRITERIA FOR DESIGN OF RCC STAGING FOR OVERHEAD WATER TANKS

0. FOREWORD

0.1 This Indian Standard was adopted by the Indian Standards Institution on 29 November 1985, after the draft finalized by the Criteria for Design of Structures Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Water tanks are important public utility and industrial structure. The design and construction methods in reinforced concrete are influenced by the prevailing construction practices, the physical property of the material and the climatic conditions. At present there is no national standard for the design of overhead tanks. This code is being published in order to fulfil the need and to lay down uniform requirements of water towers.

0.3 While the common methods of design have been covered in this code, design of structures of special forms or in unusual circumstances should be left to the judgement of the Engineer-in-charge and in such cases special systems of design and construction may be permitted on production of satisfactory evidence regarding their adequacy and safety by analysis or test or by both. Care may be taken for dust load.

0.4 In this standard it is assumed that the design of water tower is entrusted to the qualified engineer and that the execution of the work is carried out under the direction of an experienced supervisor.

0.5 The construction aspects of liquid retaining structures and design aspects of the side walls and floor of the same design aspects of the side walls and floor of the same which mainly rest on the ground level have been covered by IS : 3370 (Parts 1 to 4) * and this standard lays down the principles of design of water towers for arriving at sizes of various components of water tank.

*Code of practice for concrete structures for the storage of liquids:

- Part 1-1965 General requirements.
- Part 2-1965 Reinforced concrete structures.
- Part 3-1967 Prestressed concrete structures.
- Part 4-1967 Design tables.

0.6 All requirements of IS : 456-1978*, IS : 3370 (Part 1)-1965† and IS : 3370 (Part 2)-1965‡ is so far as they apply shall be deemed to form part of this code except where otherwise laid down in this code.

0.7 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS : 2-1960§. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

1. SCOPE

1.1 This standard lays down guidelines for layout for overhead water tanks and criteria for analysis for RCC staging both for steel and concrete tanks.

1.2 While some of the provisions of this standard in the case of RCC staging for elevated tanks, though refer to the storage of water, the recommendations are equally applicable to other materials stored.

1.3 The requirements given in this code applicable for column type staging and circular and polygonal shaft staging for overhead water tanks.

2. TERMINOLOGY

2.0 For the purpose of this standard, the following definitions shall apply.

2.1 Capacity — Capacity of the tank shall be the volume of water it can store between the designed full supply level and lowest supply level (that is, the level of the lip of the outlet pipe). Due allowance shall be made for plastering the tank from inside if any when calculating the capacity of tank.

2.2 Height of Staging — Height of staging is the difference between the lowest supply level of tank and the average ground level at the tank site.

2.3 Water Depth — Water depth in tank shall be difference of level between lowest supply level and full supply level of the tank.

*Code of practice for plain and reinforced concrete (*third revision*).

†Code of practice for concrete structures for the storage of liquids : Part 1 General requirements.

‡Code of practice for concrete structures for the storage of liquids : Part 2 Reinforced concrete structures.

§Rules for rounding off numerical values (*revised*).

3. LOADS

3.1 Dead Loads — Dead loads shall be calculated on the basis of unit weights taken in accordance with IS : 1911-1967*. Unless more accurate calculations are warranted, the unit weight of reinforced concrete made with sand and gravel or crushed natural stone aggregate may be taken as 2 500 kg/m³. Loads due to pipings and stair cases should also be considered.

3.2 Imposed Loads — Imposed loads like live loads, snow loads and wind loads shall be in accordance with IS : 875-1964†. Weight of the water may be taken as live load for members directly containing the same. The weight of water shall be considered as dead load in the design of staging.

3.3 Wind Load — Wind load shall be applied in accordance with IS : 875-1964† while analyzing the stresses the combination shall be as follows:

- a) wind load with tank empty; and
- b) wind load with tank full.

The worst combination of the stress on account of the above shall be considered while working out the permissible stresses.

3.4 Seismic Forces — When seismic loading is considered, following two cases may be considered :

- a) tank empty; and
- b) tank full.

In addition, wherever required the effect of surge due to wave formation of the water may be considered. The seismic force acting on the support for the tank and its analysis shall be in accordance with IS : 1893-1975‡.

3.5 Vibration Forces — Vibration forces such as due to blast forces (see IS : 6922-1973§) as experienced in mines, collaries and in the close proximity of railway tracks shall be considered in the design (see also 7.1.1).

NOTE — In areas of construction, where vibration forces are expected during construction for walls, reinforcement shall be provided on both faces to ensure proper bonding of the reinforcement.

*Schedule of unit weights of building materials (*first revision*).

†Code of practice for structural safety of buildings : Loading standards (*revised*).

‡Criteria for earthquake resistant design of structures (*third revision*).

§Criteria for safety and design of structures subject to underground blast.

4. ANALYSIS

4.1 General — Provision shall be made for conditions of stresses that may occur in accordance with principles of mechanics, recognized methods of design and sound engineering practice. In particular, adequate consideration shall be given to the effects of monolithic construction in the assessment of bending moment and shear.

4.1.1 Before taking up the design, the designer should first decide the most suitable type of staging of tanks and correct estimation of loads including statical equilibrium of structure particularly in regard to overturning of overhanging members shall be made. The design should be based on the worst possible combination of loads, moments and shears arising from vertical loads and horizontal loads acting in any direction when the tank is full as well as empty.

4.2 Loading Combination — Loading combination may be considered as given in IS : 456-1978* and IS : 875-1964†. Both conditions, tank full and tank empty shall be considered.

NOTE — Wherever required the effect of wave action of water on the sides of the tank may be considered.

5. BASIS OF DESIGN FOR REINFORCED CONCRETE MEMBERS

5.1 Staging and other reinforced concrete members including foundation shall be designed in accordance with the requirements of IS : 456-1978*.

5.2 Permissible Stresses — The permissible stresses in the concrete and steel reinforcement shall be in accordance with IS : 456-1978* for column staging.

5.3 Seismic Effect — Earthquake resistant design of water tanks shall be carried out in accordance with IS : 1893-1975‡.

5.4 Increase in Permissible Stresses — Increase in permissible stresses for column staging shall be as per IS : 456-1978*.

5.4.1 The increase in permissible stresses as per 5.4 need not be allowed in the design of braces for forces as wind or earthquake which are primary forces in them.

*Code of practice for plain and reinforced concrete (*fourth revision*).

†Code of practice for structural safety of buildings : Loading standards (*revised*).

‡Criteria for earthquake resistant design of structures (*third revision*).

6. LAYOUT OF OVERHEAD TANKS

6.0 Generally the shape and size of elevated concrete tanks for economical design depends upon the functional requirements such as:

- a) Maximum depth for water;
- b) Height of staging;
- c) Allowable bearing capacity of foundation strata and type of foundation suitable;
- d) Capacity of tank; and
- e) Other site conditions.

6.1 Classification and Layout of Elevated Tanks — Based on the capacities of the tank, the possible classification for types of elevated tanks may be as followed as given in **6.1.1** to **6.1.4** for general guidance.

6.1.1 For tank up to 50 m³ capacity may be square or circular in shape and supported on staging three or four columns.

6.1.2 Tanks of capacity above 50 m³ and up to 200 m³ may be square or circular in plan and supported on minimum four columns.

6.1.3 For capacity above 200 m³ and up to 800 m³ the tank may be square, rectangular, circular or intze type tank. The number of columns to be adopted shall be decided based on the column spacing which normally lies between 3.6 and 4.5 m. For circular, intze or conical tanks, a shaft supporting structures may be provided.

6.1.4 Different shapes of water towers with certain arrangements of bottom construction are shown in Fig. 1 to 4.

6.2 Besides the general shapes given in **6.1.1** to **6.1.4**, tanks of unusual shapes, such as spherical, conical or multicell may also be adopted depending upon the discretion of the designer.

7. ANALYSIS OF STAGING

7.1 Staging Components

7.1.1 Columns

7.1.1.1 Forces and moments on columns — The entire load of the tanks shall be considered to be transferred to the columns in the manner in which the floor of the tank contributes to each column. The effects of continuity of the beam at the top of columns, if any, shall be accounted for in calculating the reactions on columns. In addition to tank load, axial forces, forces

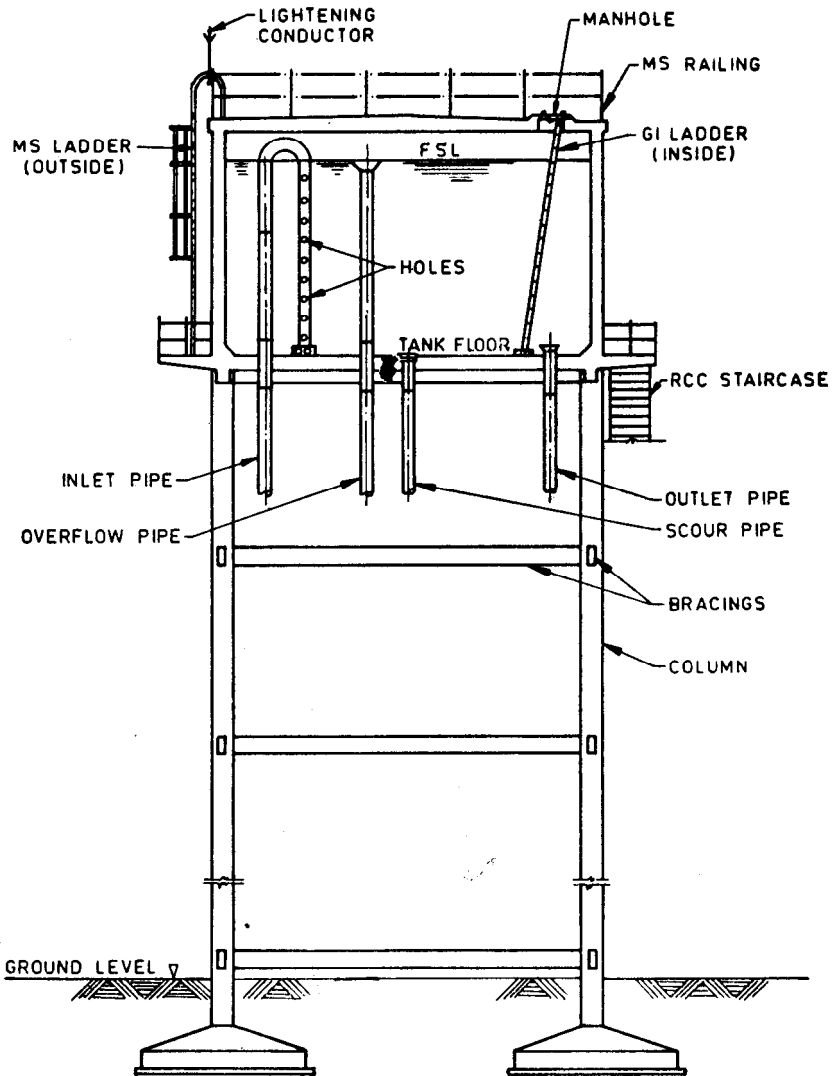


FIG. 1 TYPICAL SECTION OF A SMALL SIZED WATER TOWER

and bending moments due to wind, earthquake or vibration shall be considered in accordance with 7.2.1 to 7.2.4. In absence of more exact analysis, the effective length of columns in the evaluation of stress, may be obtained as per the recommendations given in Fig. 5.

NOTE 1 -- In places where continuous vibration is likely to occur like near railway tracks, steel staging may be preferred.

NOTE 2 — Whenever tanks are provided near mines, in addition to vibration forces, mining subsidence should also be considered.

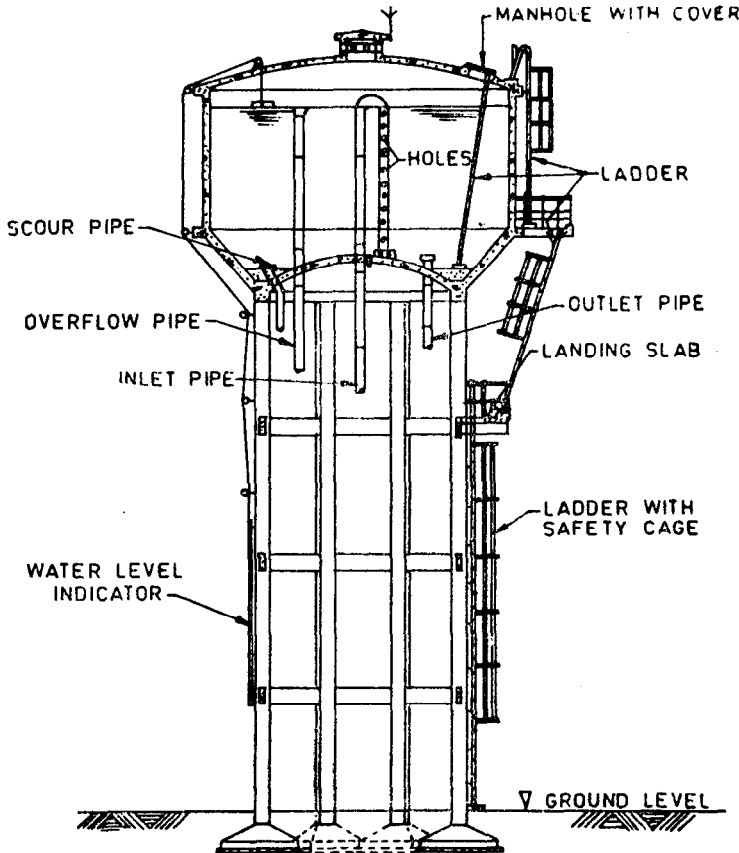
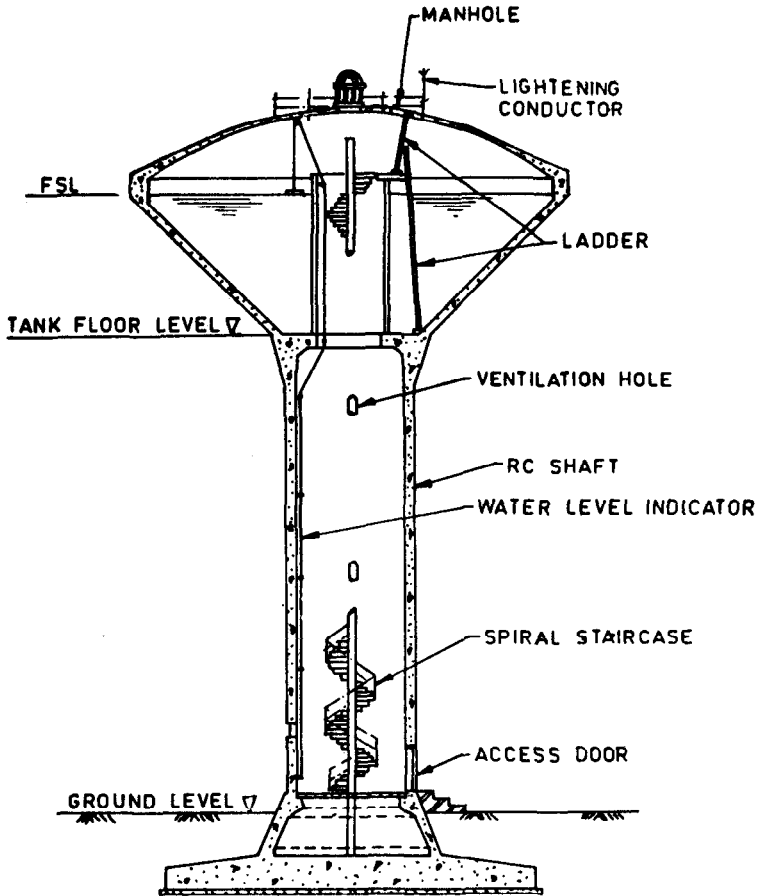


FIG. 2 TYPICAL SECTION OF A INTZE TYPE WATER TOWER

7.1.1.2 *Horizontal forces* — Actual forces and moments resulting from horizontal forces may be calculated for the critical direction and used in the design of the structure. Analysis may be done by any of the accepted methods including considering it as space frame.



NOTE — Arrangement of inlet and outlet pipes, scour and overflow pipes are similar to those shown in Fig. 1.

FIG. 3 TYPICAL SECTION OF A WATER TOWER WITH RC SHAFT

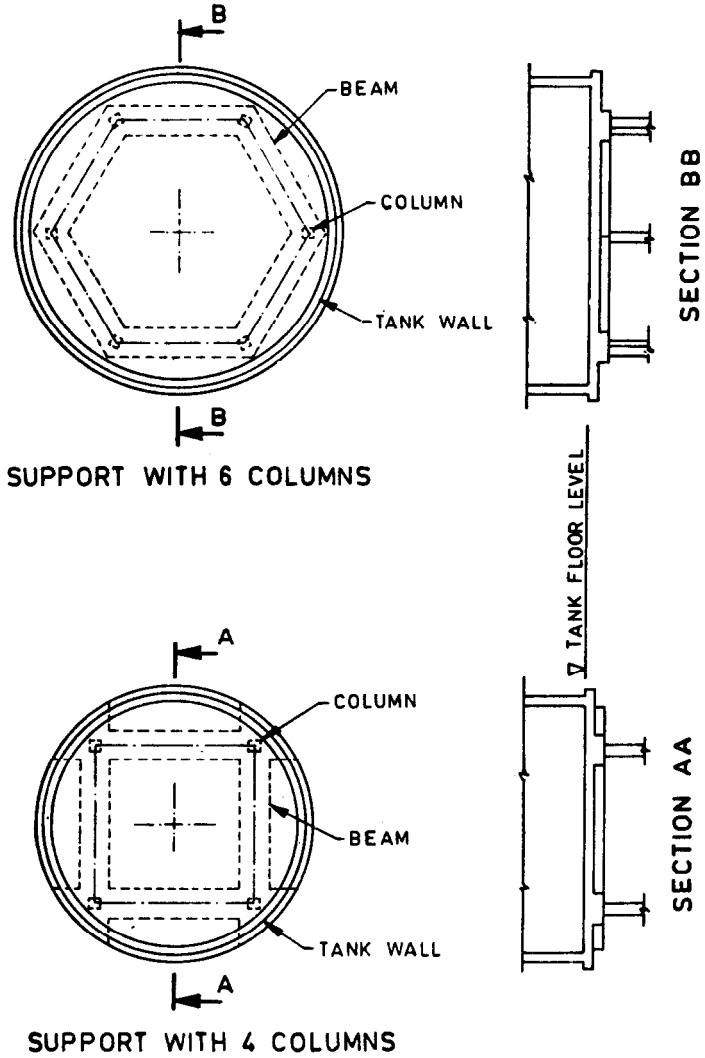


FIG. 4 TYPICAL ARRANGEMENT OF SUPPORT FOR WATER TOWER BOTTOM SLABS (Continued)

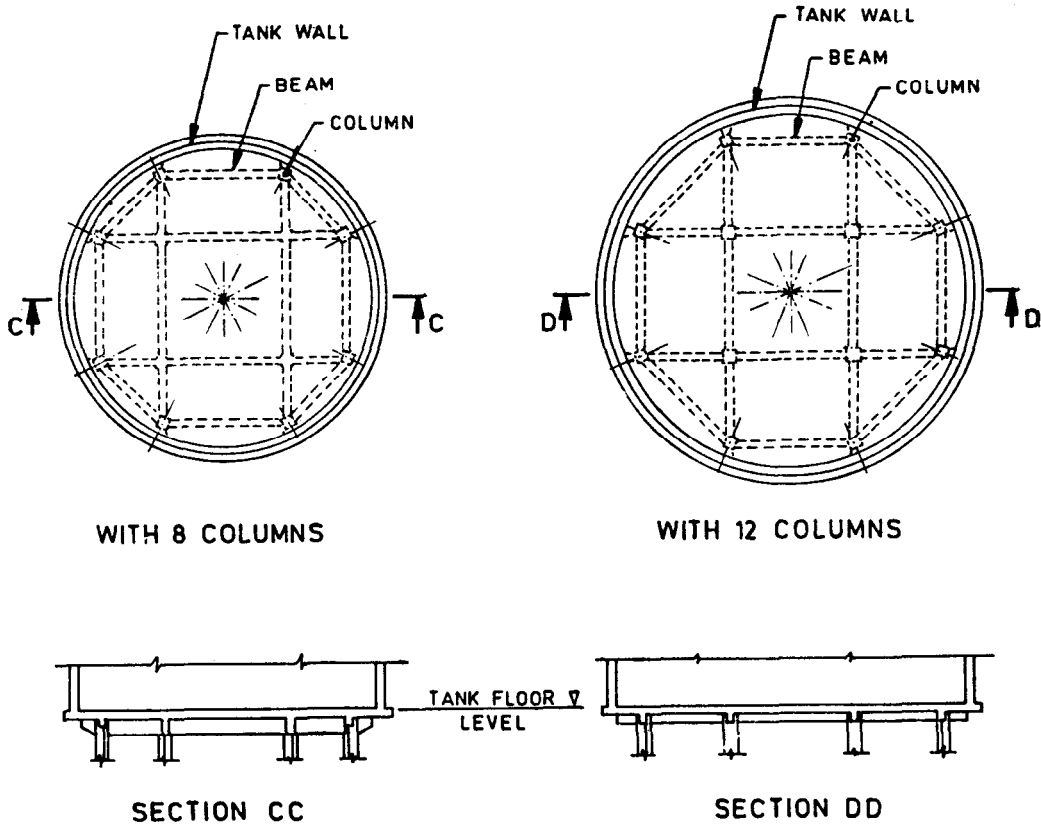


FIG. 4 TYPICAL ARRANGEMENT OF SUPPORT FOR WATER TOWER BOTTOM SLABS (*Continued*)

7.1.1.3 Horizontal forces shall be assumed to act on all parts of the tank as well as the tower. Axial forces in columns, due to horizontal loads shall be calculated by equating the moments due to all horizontal forces above the level of consideration to the restraining moment offered by axial forces in columns.

7.2 Bracings

7.2.1 For staging of height above foundation greater than 6 m, the column shall be rigidly connected by horizontal bracings suitably spaced vertically at distances not exceeding 6 m.

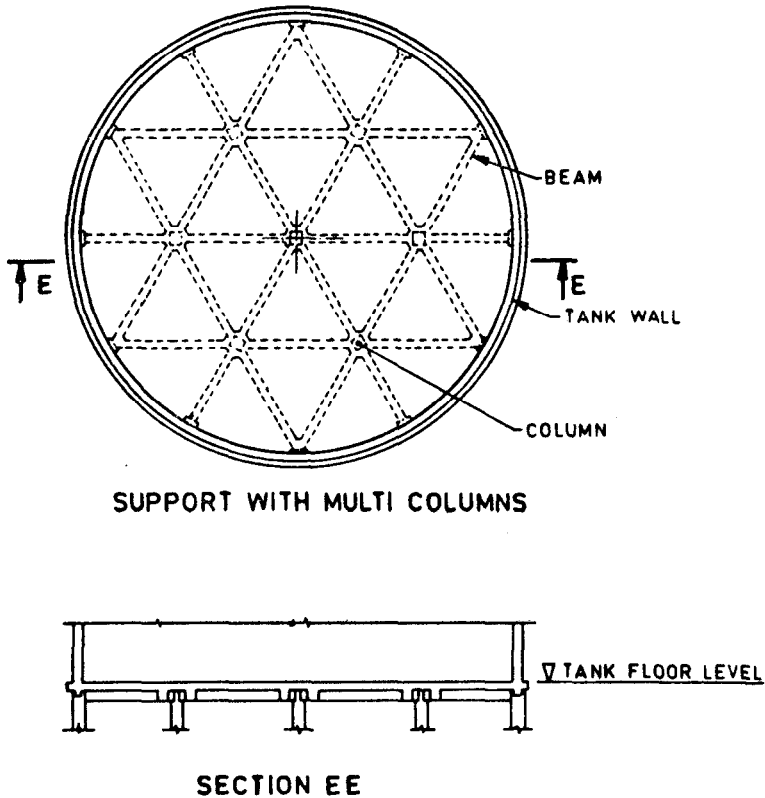


FIG. 4 TYPICAL ARRANGEMENT OF SUPPORT FOR WATER TOWER BOTTOM SLABS (Continued)

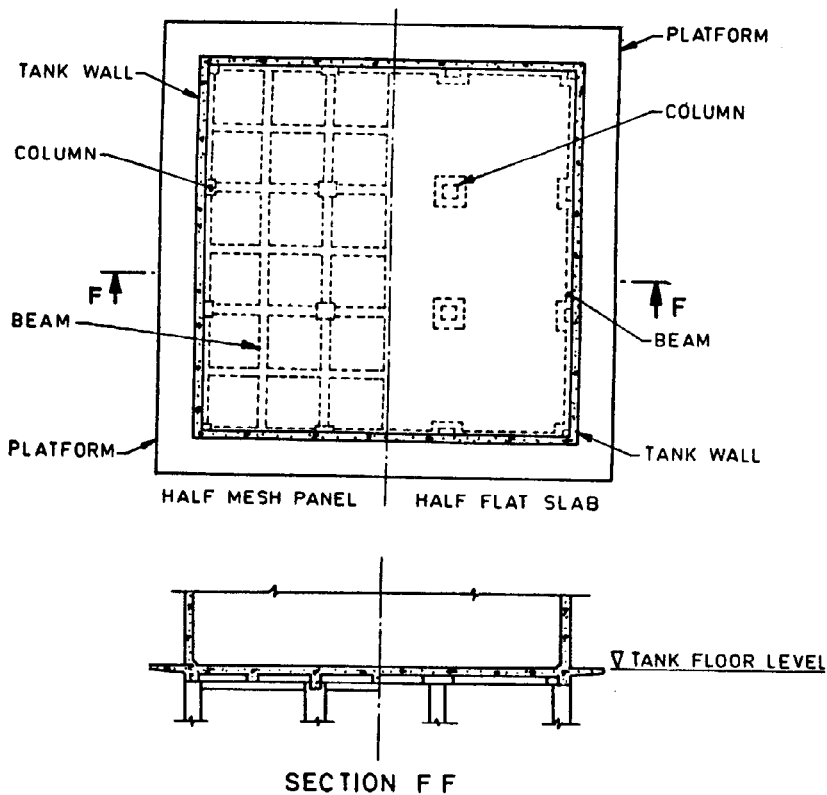
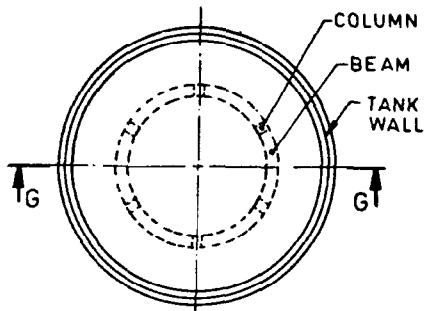
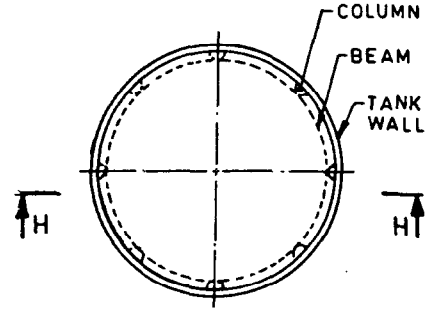


FIG. 4 TYPICAL ARRANGEMENT OF SUPPORT FOR WATER TOWER BOTTOM SLABS (Continued)

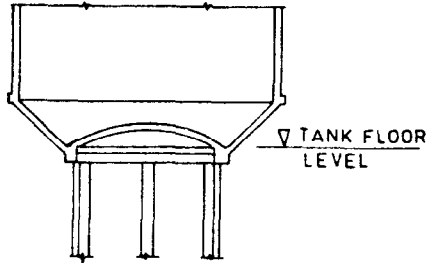


TANK WITH CIRCULAR BEAM
AT MIDDLE OF THE BASE

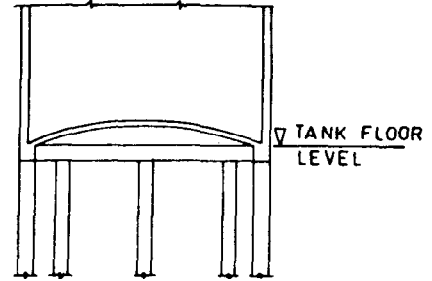


TANK WITH CIRCULAR
BEAM AT PERIPHERY

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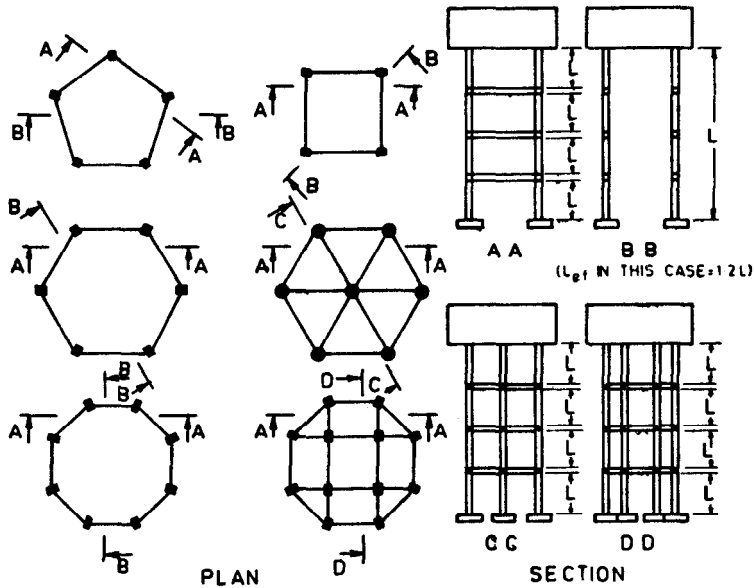


SECTION GG



SECTION HH

FIG. 4 TYPICAL ARRANGEMENT OF SUPPORT FOR WATER TOWER BOTTOM SLABS



Notes —

1. L = Unsupported length.
2. L_{eff} = Effective length to be calculated as per Appendix D of IS : 456-1978 considering without restraint against sway.
3. Stiffness of members K_c and K_b are to be calculated in the plane of bending considered.
4. Torsional stiffness of beams in other planes ignored.
5. Minimum L_{eff} in all cases to be not less than $1.2L$.

FIG. 5 EFFECTIVE LENGTH OF COLUMNS SUPPORTING WATER TANK

7.2.2 Bending moments in horizontal braces due to horizontal loads shall be calculated when horizontal forces act in a critical direction. The moments in braces shall be the sum of moments in the upper and lower columns at the joint resolved in the direction of horizontal braces.

7.2.3 Moments and shears arising from local vertical loading, if any, shall be accounted for in the design.

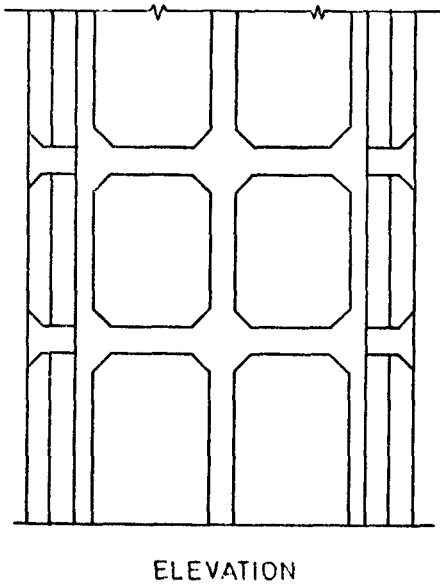
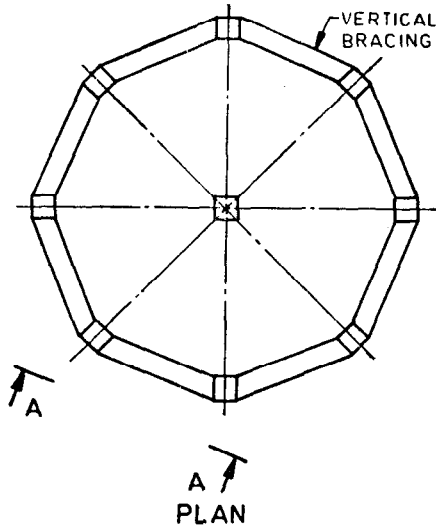
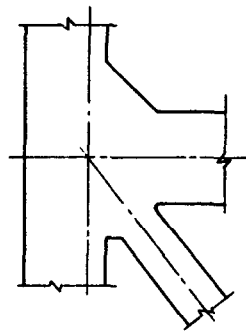
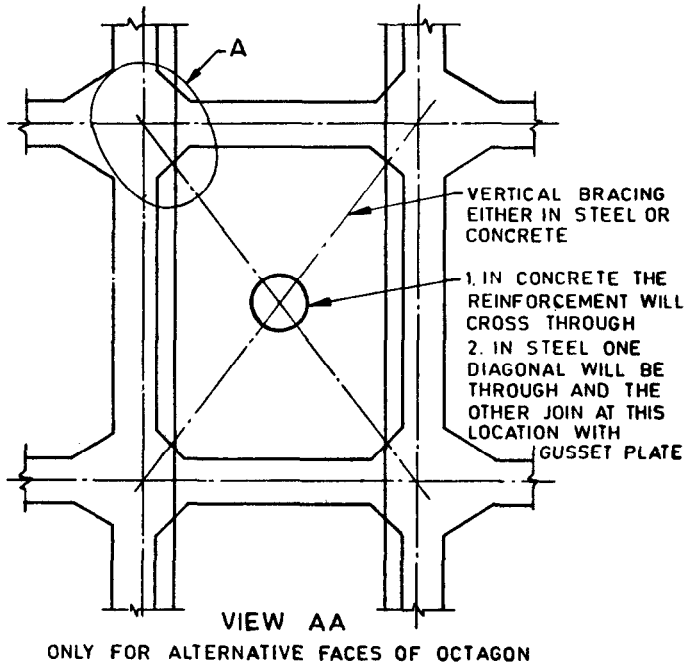


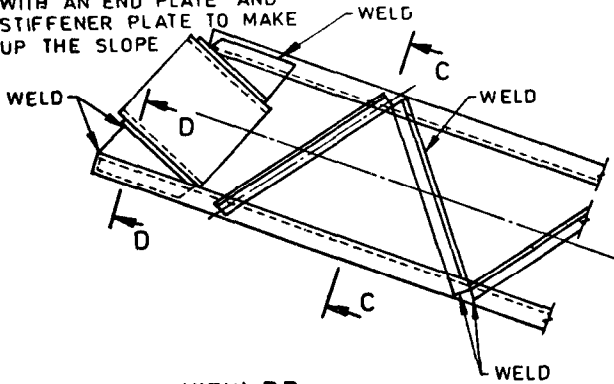
FIG. 6 TYPICAL DETAILS OF DIAGONAL BRACING OF CONCRETE OR STEEL (*Continued*)



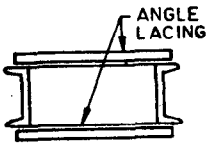
DETAIL AT A
DETAILS OF VERTICAL BRACING IN CONCRETE

FIG. 6 TYPICAL DETAILS OF DIAGONAL BRACING OF
CONCRETE OR STEEL (*Continued*)

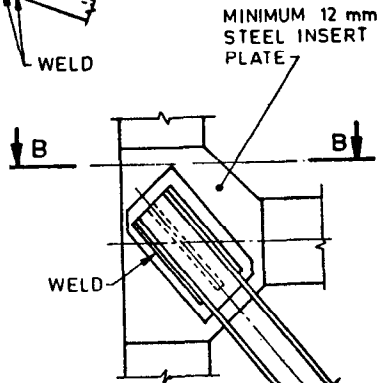
THE GAP SHALL BE MADE UP WITH AN END PLATE AND STIFFENER PLATE TO MAKE UP THE SLOPE



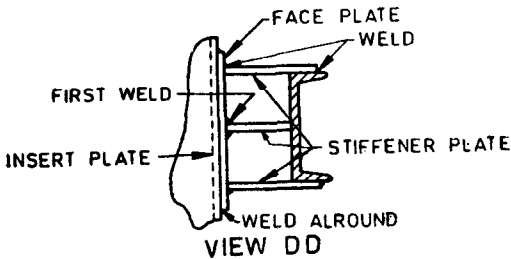
VIEW BB



VIEW CC



DETAILS OF VERTICAL BRACING IN STEEL



VIEW DD

FIG. 6 TYPICAL DETAILS OF DIAGONAL BRACING OF CONCRETE OR STEEL

7.2.4 For staging in seismic zones where seismic co-efficient exceeds 0.05 twin diagonal vertical bracing of steel or RCC in addition to the horizontal bracing may be provided (*see* IS : 4326-1976*). The typical sketch of diagonal vertical bracing is shown in Fig. 6.

7.3 Column Foundations

7.3.1 Separate footings may be provided for column staging and designed as per requirements of IS : 456-1978†. Combined footing with or without tie beam or raft foundation in accordance with IS : 2950-1965‡ may be provided. For all towers situated in seismic zones where seismic co-efficient is above 0.05, at base of the tower all columns shall be tied together above foundation level and well within ground by a structural member such as braces or ring beam. Alternately continuous strip (or annular strip) foundation, mat or raft foundations shall be provided.

7.3.2 The foundation shall be so proportioned that under vertical loads of tower (with tank full as well as empty) and effects of horizontal forces, the pressure on the soil is within the safe bearing capacity and in the critical direction the footing does not lift up at any point.

7.3.3 Loss of contact in the soil under footing should not be allowed. In locations where the soil bearing capacity is high, loss of contact may be allowed provided it is safe against overturning and such other conditions that are to be fulfilled.

7.4 Stair-Case — Access to the tank shall be provided by means of steel ladder or RCC stairs with landings adequately tied to the staging. In case of vertical ladders safety cages shall be provided for heights exceeding 6 m. In such cases cages shall start at the level of 3 m.

8. SHAFT TYPE STAGING

8.1 The tower may be in the form of single shaft circular or polygonal in plan and may be tapering. The area enclosed with the shafts may be used for providing the pipes, stairs, electrical control panels, etc.

8.2 Circular Shafts Staging

8.2.1 The minimum thickness of concrete shell for staging shall be 150 mm. When internal diameter exceeds 6 m, the minimum thickness in mm shall be $150 + \frac{D - 6000}{120}$ where D is the internal diameter of concrete shell in mm.

*Code of practice for earthquake resistant construction of buildings (*first revision*).

†Code of practice for plain and reinforced concrete (*third revision*).

‡Code of practice for design and construction of raft foundations.

8.2.2 Reinforcement in Shell

8.2.2.1 Vertical reinforcement — The minimum vertical reinforcement shall be 0.25 percent for deformed bars, of the concrete area of the section under consideration. This reinforcement shall be provided in two layers, one near each face to make up the minimum reinforcement.

The minimum diameter of longitudinal bars shall be 10 mm and the maximum centre-to-centre distance of reinforcement shall not exceed twice the thickness of shell nor 400 mm centre-to-centre in each layer.

8.2.2.2 Circumferential reinforcement — The circumferential reinforcement shall not be less than 0.2 percent, of the concrete area in vertical section under consideration subject to a minimum of 4 cm² per metre height. If the vertical reinforcement is provided in two layers, the circumferential reinforcement shall be provided in two layers and minimum reinforcement specified above shall be divided equally in each layer. The spacing of bars shall not be more than 300 mm or the shell thickness whichever is less. Circumferential reinforcement shall be placed nearer the faces of the shell.

8.2.2.3 The detailing of shaft at the opening shall take into consideration provision of effective continuity in the reinforcement above, at the side of and below the opening.

At both the top and bottom of each opening, additional reinforcement shall be placed having an area at least equal to one-half the area of the established design circumferential reinforcement interrupted by the opening, and shall extend beyond the opening to a sufficient distance to develop the bond in bars. This steel shall be placed within a height not exceeding concrete thickness at opening.

At both sides of each opening, additional vertical reinforcement shall be placed having an area at least equal to one-half the area of the established design vertical reinforcement interrupted by the opening, and shall extend beyond the opening to a sufficient distance to develop bond in bars.

Diagonal reinforcement with the total cross-sectional area in cm² of not less than half the shell thickness in cm shall be placed at each corner of the opening. Such diagonal bars shall extend past their intersection points up to a distance sufficient to develop the required bond. If the opening does not interrupt vertical or circumferential reinforcement, diagonal reinforcement may not be provided. However, provision of a nominal diagonal reinforcement to ensure crack control is advisable. For smaller opening less than 0.5 m width the diagonal reinforcement may be half the value recommended above.

8.2.3 The minimum clear concrete cover over the reinforcement shall be 40 mm for the outer side and 25 mm for the inner side of the shaft.

8.2.4 Analysis for Shaft Staging

8.2.4.1 When the wind is distributed over the circumference at the cross-section of the shaft the approximate ring moments produced are given below:

$$Moe \text{ or } Moi = 3.3 W_p \times r^2 \text{ in kg m per m height of shaft}$$

where

Moe = Local external moment produced by ovaling at external face,

Moi = Local external moment produced by ovaling at internal face,

W_p = Wind pressure at any level in N/mm², and

r = Mean radius of ring of the concrete shell in mm.

The hoop force and shear due to ovaling may be neglected.

NOTE — As long as tensile stresses due to the ring moment is within the permissible values of circumferential tensile stress in concrete no additional reinforcement is necessary. Beyond this limit reinforcement shall be provided in two layers.

8.2.5 Formulae — (For vertical stress in circular shaft).

8.2.5.1 The whole section is under compression, if :

a) for annular sections

$$\frac{e}{r} \leq \frac{1}{2} \quad \dots \dots \dots (1)$$

In such cases the maximum vertical compressive stress in concrete is given by:

$$\sigma_{cv} = \frac{W}{2 \pi r t} \left(1 + \frac{2e}{r} \right) \quad \dots \dots \dots (2)$$

b) for annular section with one opening:

$$\frac{e}{r} \leq \frac{1}{2(\pi - \beta)} \left[\frac{(\pi - \beta)^2 - \sin^2 \beta}{(\pi - \beta) \cos \beta + \sin \beta} - 3 \sin \beta \right] \quad \dots \dots \dots (3)$$

then in such cases, the maximum vertical compressive stress is given by:

$$\sigma_{cv} = \frac{W}{2(\pi - \beta)rt} \left[1 + \frac{2 \left\{ \frac{e}{r} + \frac{\sin \beta}{(\pi - \beta)} \right\} \{ (\pi - \beta) \cos \beta + \sin \beta \}}{(\pi - \beta) - \frac{1}{2} \sin 2\beta - \frac{2 \sin^2 \beta}{(\pi - \beta)}} \right] \dots \dots \dots (4)$$

where

$$e = \frac{M}{W} = \frac{\text{Moment in vertical plane at the section under consideration in N mm}}{\text{Total vertical load above section under consideration in N}}$$

r = Mean radius of circular section under consideration in mm;

t = thickness of shaft shell at section under consideration in mm;

β = Half the angle subtended by the neutral axis as a chord on the circle of radius r , in degrees unless otherwise specified; and

σ_{cv} = Maximum vertical stress in concrete at outside diameter of shaft shell in N/mm².

8.2.5.2 If e/r is greater than the corresponding right hand side of expressions (1) or (3) of 8.2.5.1, then α defining the position of neutral axis may be calculated from the general expression (5) by trial (see Fig. 7).

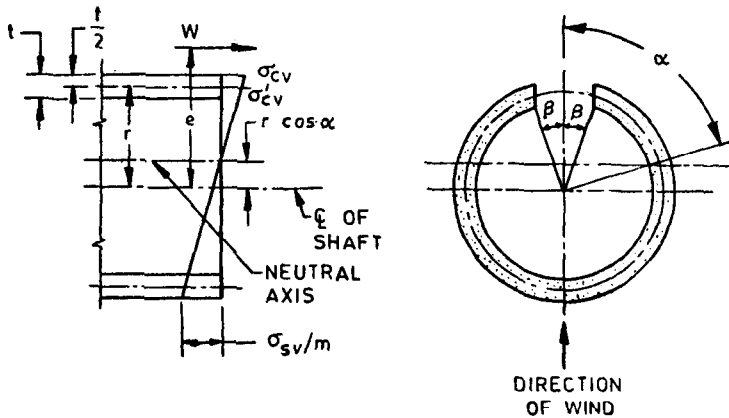


FIG. 7 SHAFT WITH OPENING

a) Put $\beta = 0$, for annular section without opening

where

$\alpha =$ one half the central angle subtended by neutral axis as a chord on the circle of radius r , in degrees unless otherwise specified.

$$\frac{e}{r} = \left(\frac{A}{B} \right) \quad \dots \dots \dots (5)$$

where

$$A = \frac{1}{2} (1-p) (\alpha - \sin \alpha \cos \alpha) - \frac{1}{2} (1-p + mp) (\beta + \sin \beta \cos \beta - 2 \cos \alpha \sin \beta) + \frac{1}{2} m\pi p; \text{ and}$$

$$B = (1-p) (\sin \alpha - \alpha \cos \alpha) - (1-p + mp) (\sin \beta - \beta \cos \alpha) - mp\pi \cos \alpha$$

where

$m =$ modular ratio; and

$p =$ ratio of total area of vertical reinforcement to total area of crete of shaft shell at section under consideration.

8.2.5.3 The maximum vertical compressive stress in concrete due to combined effect of vertical loads and lateral wind loads, σ_{cv} may be concalculated by:

$$\sigma_{cv} = \sigma_{cv}' \left[1 + \frac{t}{2r \cos \beta (\cos \beta - \cos \alpha)} \right] \quad \dots \dots \dots (6)$$

where

$$\sigma_{cv}' = \frac{W}{2rt} \left[\frac{(\cos \beta - \cos \alpha)}{(1-p) (\sin \alpha - \alpha \cos \alpha) - (1-p + mp) (\sin \beta - \beta \cos \alpha) - mp\pi \cos \alpha} \right] \quad \dots \dots \dots (7)$$

8.2.6 Permissible Stresses

8.2.6.1 The stress in concrete shall not exceed the following limits for various combination of loads:

<i>Combination Load</i>	<i>Stress Limit</i>
a) Dead load + wind load	0.38 σ_{cv}
b) Dead load + earthquake forces	0.40 σ_{cv}
c) Circumferential tensile stress in concrete due to wind induced ring moment	0.07 σ_{cv}

where

$\sigma_{cv} =$ 28-day ultimate cube strength of concrete in N/mm².

8.2.6.2 Reinforcement — The stresses in steel shall not exceed the following limits for various combination of loads

<i>Combination</i>	<i>Stress Limit</i>
a) Dead load + wind load	0.57 σ_{sy}
b) Dead load + earthquake load	0.60 σ_{sy}
c) Circumferential stress in steel due to wind induced ring moment (<i>see Note</i>)	0.50 σ_{sy}

where

σ_{sy} = yield or proof stress of steel in N/mm².

NOTE — If shell thickness is adequate to satisfy 8.2.6.1 (c), this requirement may be waived.

8.3 Eccentricity due to Settlement — The cross-section of the shaft shall also be checked for stresses resulting from eccentricity caused by the permissible differential settlement of foundation as per IS : 1904-1978*.

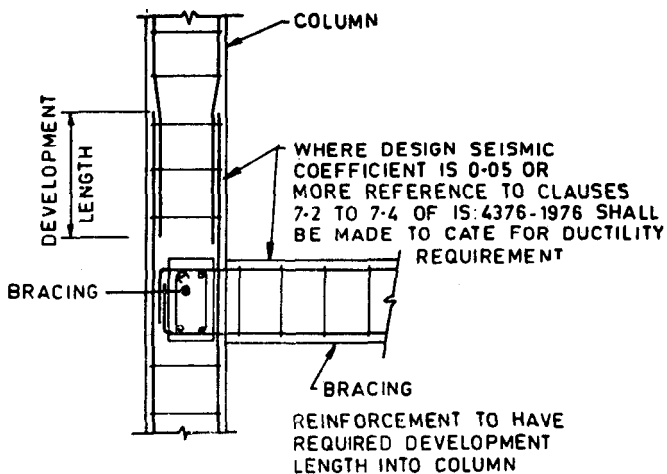
8.4 Polygonal Shaft — Polygonal shaft may be designed as a circular shaft considering equivalent diameter based on inscribed circle of the shaft.

9. DETAILING

9.1 Requirement governing detailing shall be in accordance with the relevant clause of IS : 456-1978†. Typical reinforcement details for column and bracing, shaft type staging and intz tank are given in Fig. 8 to 11.

*Code of practice for structural safety of buildings ; Shallow foundations (*second revision*).

†Code of practice for plain and reinforced concrete (*third revision*).



JUNCTION OF COLUMN
AND BRACING

FIG. 8 TYPICAL REINFORCEMENT DETAILS OF JUNCTION OF
COLUMN AND BRACING

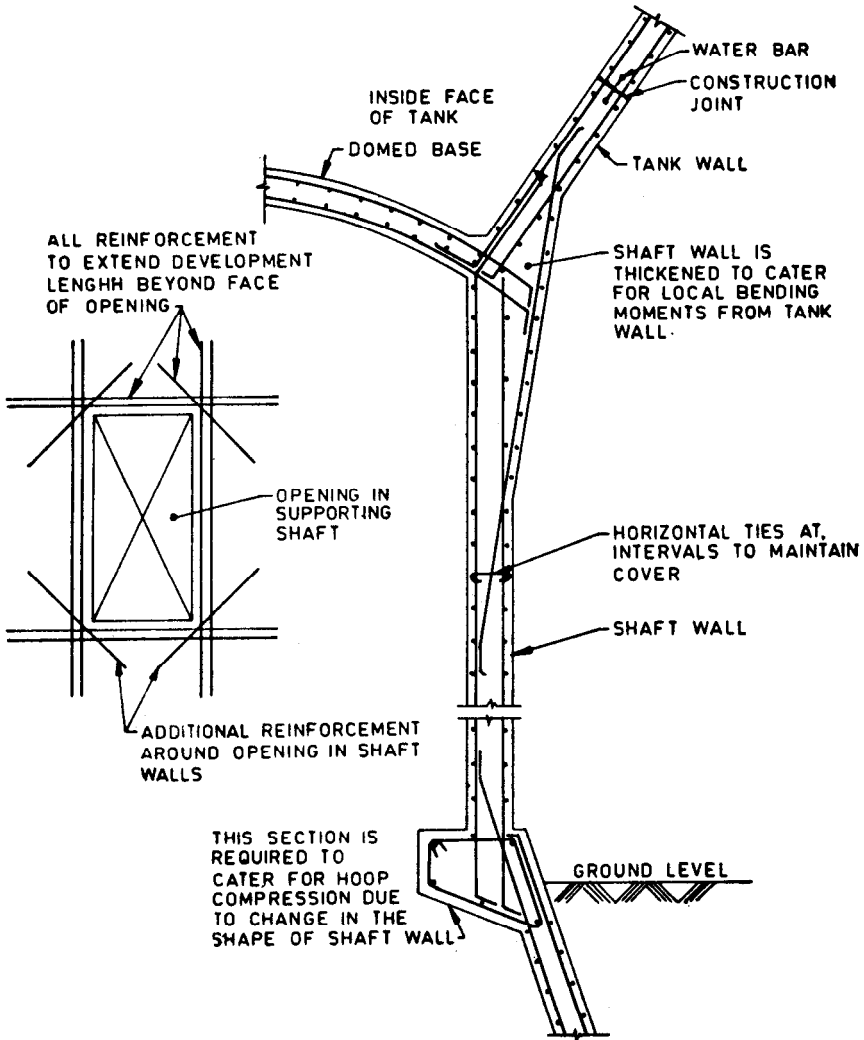


FIG. 9 TYPICAL REINFORCEMENT DETAILS OF SHAFT

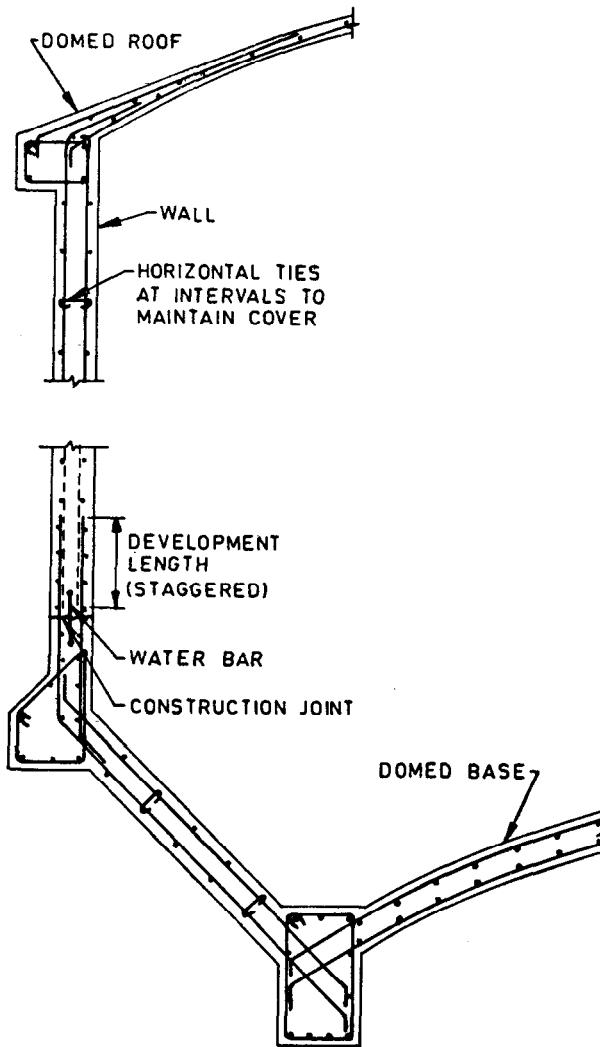
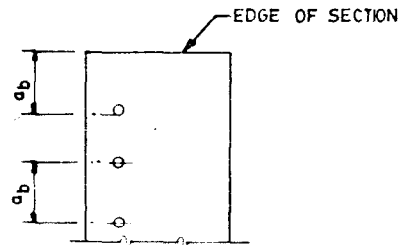
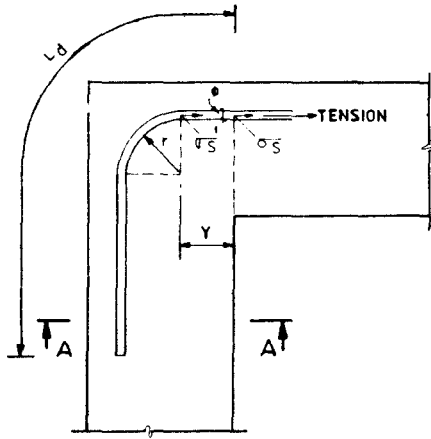


FIG. 10 TYPICAL REINFORCEMENT DETAILS OF INTZE TYPE TANK



SECTION AA

$$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}} \quad \text{REF: (CI 2521) (IS-456-1978)}$$

$$\sigma_s^i = \frac{L_d - Y}{L_d} \times \sigma_s$$

$$r = \left[\frac{\pi \sigma_s^i (1 + \frac{2\phi}{a_b})}{4 f_{ck}} \right] \times \phi \quad \text{REF: (CI.25.2 25) (IS:456-1978)}$$

L_d = DEVELOPMENT LENGTH OF BAR.

ϕ = DIA OF BAR.

σ_s = STRESS IN BAR AT THE FACE OF SECTION.

σ_s^i = STRESS IN BAR AT THE MID-POINT OF BEND.

f_{ck} = CHARACTERISTIC STRENGTH OF CONCRETE

a_b = DISTANCE BETWEEN BARS AS SHOWN.

r = INTERNAL RADIUS OF BEND.

τ_{bd} = DESIGN BOND STRESS.

NOTE: STRESSES IN THE ABOVE CALCULATION ARE BY WORKING STRESS METHOD OF DESIGN.

FIG. 11 MINIMUM INTERNAL RADIUS OF BEND FOR EFFECTIVE ANCHORAGE OF TENSILE BARS CONSIDERING LIMIT ON BEARING STRESS IN CONCRETE AT BENDS AS PER IS : 456-1978

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